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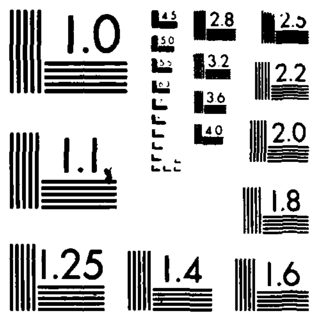

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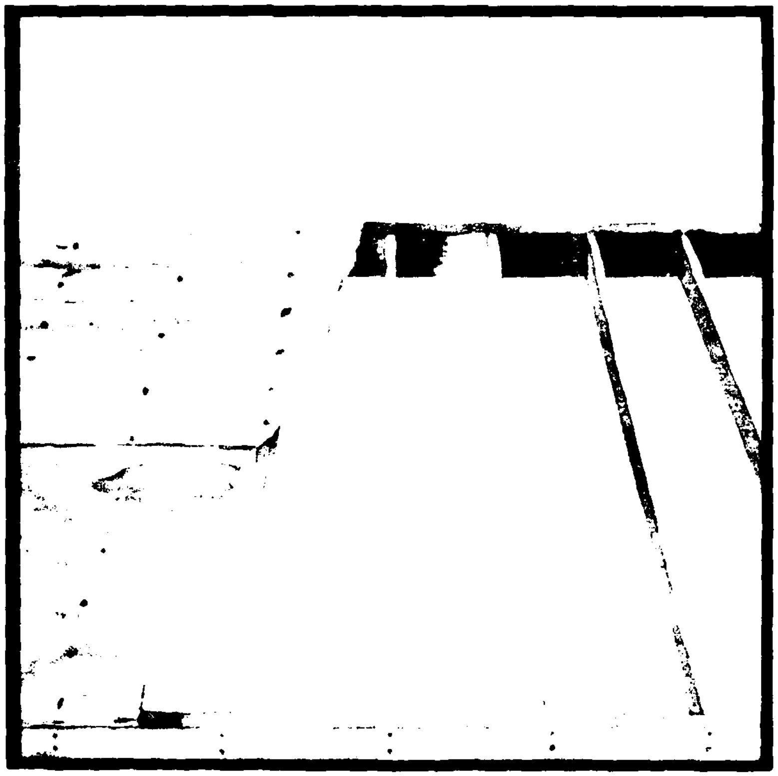
Predicting the Strength of Wood-Joist Floors

Dan L. Wheat
Russell C. Moody



Abstract

Most structural analyses of wood-joist floors have been based on the assumption of linear response. Although linear models, such as the FEAFLO computer program, adequately predict stiffness at service loads, they do not adequately predict failure loads because they do not include the nonlinear behavior of the nailed joints. Models based on nonlinear behavior are available, but they are more complicated and costly to use than linear analyses. This study used the nonlinear model NONFLO and the linear model FEAFLO to explore relationships between results of linear and nonlinear analyses.

NONFLO predicted that the first joists to fail in 72 floors of one design would fail at about 91 percent of the load predicted by FEAFLO. This adjustment factor could be used in FEAFLO when that program is used to predict failure loads for this particular floor design. Similar factors could be computed for other types of floors.

We also determined a reduced nail stiffness which could be used in FEAFLO to predict the load at first joist failure. Such a "substitute connector stiffness" could be used with linear analyses to predict strengths nearly as accurately as those predictions made by nonlinear analyses, and at less cost.

This information will enable researchers and organizations which wish to assess the structural reliability of wood floor systems to use less expensive linear analysis techniques to account for the nonlinear behavior of wood-joist floors.

Keywords: Floors, wood joist, structural analysis, nonlinear models, strength, stiffness, light-frame construction.

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Preface

This study was intended to explore the relationships between strengths at first rupture predicted by linear and nonlinear models for wood-joist floors of a given configuration but having various combinations of joist material properties. The study produced two ways of altering linear models to account for the nonlinear behavior of nails in floors loaded to failure. The strength at first joist failure of 72 floors was analyzed using linear and nonlinear finite-element models and statistical comparisons.

The results of the study are presented in two forms. The first is recommended factors by which the predicted failure loads from the linear analysis can be scaled to give failure loads consistent with the nonlinear analysis. The second is recommended substitute nail stiffnesses (~~Wheat and others 1983~~) which can be used in a linear analysis to predict floor responses with near-equivalent accuracy as provided by a nonlinear analysis. The results presented are specific to the floor design used in this study, but similar techniques could be used to derive equivalent information for other floor designs.

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Introduction

Light-frame wood-joist floors are complex structures, and interactions among the parts of such floors are as complex as those in any structure. A full understanding of these interactions would lead to more efficient use of material and meet the needs of the user.

The stiffnesses of various parts of the floor and their contributions to the stiffness of the floor as a whole are an important set of interactions in determining the structural performance of such systems. These interactions are nearly linear at service loads; thus linear models now used, such as FEAFLO (Finite-Element Analysis of FLOors) (Thompson and others 1977), are useful in predicting the performance of floors within the service load range. However, linear models are less useful in predicting the performance of floors under greater loads. At higher loads, floor components behave nonlinearly, causing FEAFLO to underestimate deflections, overestimate connector forces, and overestimate predicted failure loads.

The apparent loss of stiffness at higher loads can be attributed to loss of stiffness in one or more of the constituent parts of the floor: connectors (e.g. nails or glue), joists, or sheathing. At high load levels, creep also produces a nonlinear load-deflection response. The analysis described in this report is concerned with connector stiffness and its influence on the behavior of wood-joist floors loaded to failure (the first occurrence of a joist rupture). From a design standpoint, this failure might be termed a "strength at first rupture" limit state.

Nonlinear models, such as NONFLO (Wheat and others 1983), take some or all of these nonlinear responses into account better than do linear models in predicting responses at high loads. Unfortunately, nonlinear models are complex and expensive to use. In this study, we have used NONFLO to derive adjustment factors for one design of floor that can be used in FEAFLO to increase FEAFLO's accuracy at high loads. The factors derived in this study are (a) direct adjustment of the failure load predicted by FEAFLO using nail stiffnesses normally associated with the service load level and (b) reduced nail stiffness to be used in FEAFLO to directly estimate the failure load. These two factors are used independently. Use of such methods to improve the accuracy of linear models at high loads should enable researchers to explore high-load responses more quickly and economically than is possible with nonlinear models.

The research in this study used one option of the computer program NONFLO to accommodate the nonlinear load-slip characteristics of the nails. This option is a nonlinear analysis with a searching algorithm to determine the failure load while simultaneously satisfying equilibrium and compatibility at impending failure. NONFLO has been verified by experimental studies (Wheat and others 1983) and used to perform pilot studies of the effects of connector nonlinearity on predicted failure loads, plus other parametric studies.

Methodology

The following methodology was used to achieve the objective of this research:

1. Determine the floor configuration to be studied, the number of floors of this configuration to be analyzed, and the material property data of each floor.
2. Select an appropriate load-slip curve to represent the nail behavior. The computer program NONFLO can accommodate any slip curve, provided it is a monotonic, continuous function which passes through the origin and is differentiable there.
3. Analyze each floor using option 3 of NONFLO to determine the predicted failure load, maximum nail forces in each joist, maximum deflections at impending failure for each joist, and the distribution of nail secant stiffnesses throughout the floor at impending failure.
4. Using the predicted failure loads from NONFLO and the corresponding predicted failure loads from FEAFLO, relate the differences throughout the range of predicted failure loads (the linear analyses using FEAFLO were obtained from another study (Vanderbilt and others 1983)).
5. Recommend substitute nail stiffnesses, based on the nail secant stiffness data obtained in step 3, to be used in a linear analysis to provide near-equivalent floor responses as predicted from the nonlinear analysis.
6. Compare the deflections and nail forces predicted from linear and nonlinear analyses throughout the range of minimum to maximum failure loads.

We tried to develop the simplest possible relationship between the predicted failure loads from linear and nonlinear analyses.

Selection of Floors and Materials

The floor configuration selected (fig. 1) is the same as used by Vanderbilt and others (1983) in the linear analyses. Each floor contained 10 nominal 2- by 8-inch joists, which were free to deflect, plus one joist on each side of the floor supported throughout its length. Joist spacings were 16 inches, and the span was either 12 feet 10 inches or 13 feet 1 inch depending on whether southern pine or Douglas-fir species groups were used. All boundaries in the finite-element analyses were assumed as simply supported.

The material properties of the joists and sheathing used as input to NONFLO are described in the following sections. These properties include the modulus of elasticity (MOE) and modulus of rupture (MOR) of the joists and the effective axial and bending MOE of the sheathing (both parallel and perpendicular to the face grain). All the nails in a given floor were assumed to behave according to the same nonlinear force-slip equation which was programmed into NONFLO.

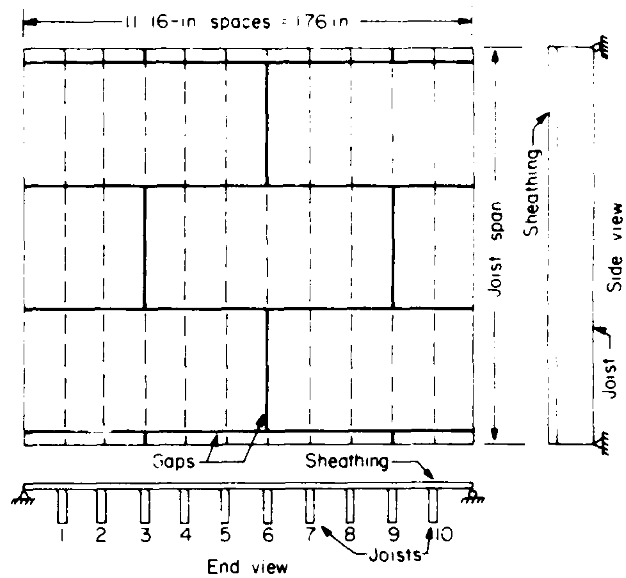

Figure 1.—Floor configuration. (ML83 5448)

Joist Properties

The joist strength and stiffness data used in this study and in Vanderbilt and others (1983) were obtained from an "in-grade" testing program carried out cooperatively by the Forest Products Laboratory (FPL), the Western Wood Products Association, the West Coast Lumber Inspection Bureau, and the Southern Pine Inspection Bureau. Field tests were conducted on mill-inventory, No. 2 Douglas-fir and southern pine material (visually graded at the mill). Data obtained on each board included its dimensions, species and grade, moisture content, MOE, and MOR. The data were collected in 10-piece serial lots where each lot was further classified as on-grade, meaning that the material was verified by the grading agencies as being of No. 2 grade, or as-graded, meaning that the material had been stamped by the mill for sale as No. 2 grade.

Selected data from this in-grade testing program were used in this study. These data include Douglas-fir (green and dry material) as-graded and on-grade lots and southern pine (dry material) as-graded and on-grade lots, or four primary material groups. Using FEAFLO to predict failure, Vanderbilt and others (1983) analyzed 107 floors (or 107 lots of 10 joists per lot) of southern pine as-graded material, 137 floors of southern pine on-grade material, 138 floors of Douglas-fir as-graded material, and 177 floors of Douglas-fir on-grade material. Ideally, it would be desirable to analyze each of the 559 floors using NONFLO, but the cost would be prohibitive. Thus, 18 floors in each of the 4 primary material groups were analyzed using NONFLO in an attempt to ascertain the computed differences between the linear and nonlinear analyses. In the finite-element analyses, no moisture content adjustments for size or strength were made.

Vanderbilt presented a cumulative distribution function (CDF) for the failure loads for all the floors of each of the four primary material groups. However, only 18 floors in each group were chosen for analysis with NONFLO. The 18 floors chosen were distributed along the CDF as follows: the 7 lowest; 4 between the 5th and 10th percentiles; and 1 each at approximately the 15th, 20th, 30th, 50th, 70th, 85th, and 95th percentiles. All subsequent references to the floors in the remaining sections of this report are based on a numbering scheme where the floors are numbered sequentially depending on their locations on the CDF, Floor 1 having the lowest failure load and Floor 18 having the highest failure load of the 18 floors chosen.

Sheathing Properties

Throughout all the analyses, the elastic properties of the plywood were held constant. Both the linear and nonlinear finite-element models (Thompson and others 1977; Wheat and others 1983) treat the floor as a series of crossing beams—partially composite T-beams in the joist direction, and sheathing beams perpendicular to the joists. This formulation neglects the twisting moments in the sheathing, so that neither an in-plane shear modulus nor Poisson's ratio is required as input. The directional properties of the plywood necessitate that the effective bending rigidity and the effective axial rigidity be input for each orthogonal direction. The effective bending rigidities for stresses induced perpendicular and parallel to the face grain of the plywood are $EI\perp$ and EI , respectively. Likewise, the effective axial rigidities for stresses induced perpendicular and parallel to grain are $EA\perp$ and EA , respectively. Specific values per foot of width are as follows:

$EI\perp$ = 106.3 k-in.^{2}
$EA\perp$ = 6525. k
EI = 287.5 k-in.^{2}
EA = 7950. k

To determine these values, we used veneer combinations that would give average values. Finally, we assumed the plywood thickness was 19/32 inch (5/8 nominal) for all floors.

Closely related to the plywood properties are the stiffnesses of the gaps between adjacent pieces of plywood. Assuming that adjacent pieces of plywood physically touch one another, this stiffness, as defined by Jizba (1978), is the MOE of the gap divided by the gap length, or E/L, and is used to simulate a relatively flexible tongue-and-groove joint or square-edged joint. The gaps perpendicular to the joists were assigned a stiffness of 5,000 lb/in.^{2} per inch of width, where E = 500 lb/in.^{2} and L = 0.1 inch, and those parallel to the joists were assigned a lower stiffness of 500 lb/in.^{2} per inch of width (Vanderbilt and others 1983).

Nail Slip Curve

A complex interaction exists between a nail and the surrounding wood fibers along their contact surface in a joint. Specific interactive forces and deformations are not treated in FEAFLO or NONFLO; rather the overall slip characteristic in the nailed joint is included through a relationship between the shear transmitted through the joint and the slip in the joint where the slip includes the deformation of the nail itself and the deformation of the surrounding wood caused by the interaction of the nail and the wood.

McLain (1976) presented the following equation to relate nail force and slip in two-member joints:

$$P = A \log_{10}(1 + B\Delta) \qquad (1)$$

where P = nail force in pounds
Δ = nail slip in inches
A, B = curve fitting constants.

McLain developed equation (1) through a nonlinear regression analysis using data from hundreds of specimens containing two members joined by a single 8d nail. It is valid for slips up to 0.10 inch. In addition, McLain presented the following equations for predicting the parameters A and B:

$$A = 227.3 - 9.813(1/\alpha_s)^2 - 2.221(1/\alpha_M)^2 \qquad (2)$$

$$B = -397.5 + 2498.3(\alpha_s)^2\alpha_M + 213.0(1/\alpha_s) + 15.83(1/\alpha_M)^2 \qquad (3)$$

where α_s = specific gravity of side member (member adjacent to the head of the nail)
α_M = specific gravity of the main member (member receiving the point of the nail).

Equation (1) is represented in figure 2, where it is seen that nail force is a monotonic function of nail slip. The curve in figure 2 is the slip behavior of a single nail. Many other slip curves have been proposed, but equation (1) was chosen for this study because it has been used successfully in comparison studies of NONFLO (Wheat and others 1983) with test data from full-scale floor tests. Equations (1) through (3) were derived from tests of two-member specimens containing species group combinations which were the same as those used in the full-scale tests of floors reported in Wheat and others (1983). McLain presents constants A and B for specific combinations of main and side members, but equations (2) and (3) allow one to predict A and B for other species group combinations for which specific gravities are known.

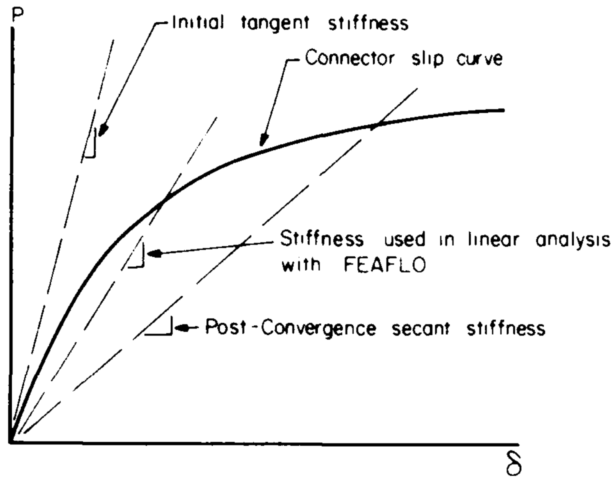

Figure 2.—Nail force-slip curve. (ML83 5449)

Nearly any nail slip curve, including equation (1), has a high value of tangent stiffness at its origin, much higher than should be used in a linear analysis even at service load levels or below. Realizing this, Vanderbilt and others (1983) obtained reduced values for use in FEAFLO where the magnitude of the connector stiffness presented by Vanderbilt depended on the species group combinations of the joined members. This is schematically represented in figure 2 as a reduced value from the initial tangent stiffness. The post-convergence secant stiffness shown in figure 2 is discussed later.

In NONFLO it is assumed that the behavior of each nail in the floor is governed by equation (1) with values of both A and B the same for all, thus implying that the factors having an influence on A and B are assumed to be the same throughout the floor.

Results

Failure Loads

Table 1 summarizes the average predicted failure loads from the linear and nonlinear analyses for the floors having joists of southern pine as-graded, southern pine on-grade, Douglas-fir as-graded, and Douglas-fir on-grade material. Details of the individual floor analyses are in Appendix A. In each case, the failure load predicted from the nonlinear analysis is less than that from the linear analysis because of the loss of stiffness in the floor with increasing load. In table 1 the ratios of the failure loads predicted from the nonlinear and linear analyses, W_{NL}/W_L, are given for the floors considered in two groups: those below the 8th percentile of the CDF (Floors 1-9) and those above the 8th percentile (Floors 10-18). The mean ratios for the 18 floors of each of the 4 primary material groups are, respectively, 0.924, 0.932, 0.902, and 0.914 for the southern pine as-graded, southern pine on-grade, Douglas-fir as-graded, and Douglas-fir on-grade. Of the 72 floors analyzed, the lowest W_{NL}/W_L ratio was 0.863 for Floor 14 of the Douglas-fir as-graded material group (Appendix A). This floor was approximately at the 30th percentile on the CDF. The highest W_{NL}/W_L ratio was 0.985 for Floor 9 of the southern pine on-grade group. This was at approximately the 8th percentile on the CDF. Because the floors analyzed using NONFLO lose stiffness with increasing applied load, the W_{NL} values tend to be smaller relative to W_L for the floors controlled by joists having higher strengths, that is, those higher on the CDF. This general trend is reflected in the results in table 1. A graphical representation of the W_{NL}/W_L ratios versus the level on the CDF is shown in figure 3 for all 72 floors.

Table 1.—Ratios of W_{NL} to W_L as a function of level on cumulative distribution function (CDF)

Grading	Mean value of W_{NL}/W_L for floors 1-9[1]	Mean value of W_{NL}/W_L for floors 10-18[1]	Row mean
	SOUTHERN PINE		
As-graded	0.93	0.92	0.92
On-grade	.95	.91	.93
	DOUGLAS-FIR		
As-graded	.92	.89	.90
On-grade	.92	.91	.91
Column mean	.93	.91	.92 = overall mean

[1]Floors 1-9 are below the 8th percentile on CDF. Floors 10-18 are from the 8th percentile to the 95th percentile on CDF.

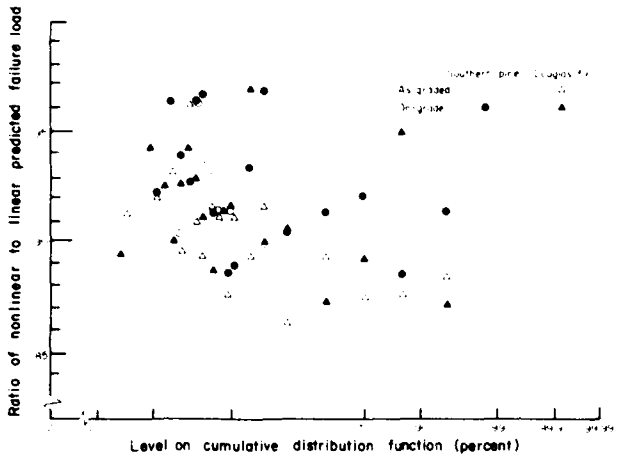

Figure 3.—Ratios of predicted failure loads in terms of the level on the cumulative distribution function. (ML83 5450)

In all cases but one (Floor 13 of the southern pine on-grade material), the controlling joist in any floor was the same in both linear and nonlinear analyses. The nonlinear analysis of Floor 13 indicated a near simultaneous failure of two joists, numbers 6 and 8, but as noted in table A-1, joist 8 controlled in the nonlinear instead of joist 6 which controlled in the linear analysis.

To further quantify the differences in the predicted failure loads between the linear and nonlinear analyses, polynomial regressions were carried out for the floors of each of the four primary material groups plus combinations of these groups as follows:

1. southern pine (as-graded), 18 data points
2. southern pine (on-grade), 18 data points
3. Douglas-fir (as-graded), 18 data points
4. Douglas-fir (on-grade), 18 data points
5. southern pine (as-graded and on-grade), 36 data points
6. Douglas-fir (as-graded and on-grade), 36 data points
7. southern pine and Douglas-fir (as-graded and on-grade), 72 data points.

We sought for each of the seven sets of data to express the failure loads predicted from the nonlinear analyses, W_{NL}, in terms of the failure loads predicted from the linear analyses, W_L, i.e. $W_{NL} = f(W_L)$ where $f(\cdot)$ is the polynomial to be determined. The following polynomials were tried for each of the seven sets of data:

$$W_{NL} = a_1 W_L$$

$$W_{NL} = a_1 W_L + a_2 W_L^2$$

$$W_{NL} = a_1 W_L + a_2 W_L^2 + a_3 W_L^3$$

In each of the seven sets the simplest polynomial, the one linear in W_L, had the highest standard deviation about the regression line and technically was the least suitable. However, the standard deviations among the seven did not vary substantially and the inclusion in the analysis of all or part of the remaining floors along the CDF could easily change the standard deviations to favor different polynomials. Thus, a reasonable and simple relationship between W_{NL} and W_L is

$$W_{NL} = aW_L \tag{4}$$

where

a = 0.92, southern pine, as-graded
0.92, southern pine, on-grade
0.89, Douglas-fir, as-graded
0.91, Douglas-fir, on-grade
0.92, southern pine, as-graded and on-grade
0.90, Douglas-fir, as-graded and on-grade
0.91, all data

Thus, according to the simple linear relationship of equation (4), FEAFLO overestimates the load at first joist rupture by 8 to 11 percent because of the assumption that the nail slip relationship is linear instead of that given by equation (1).

Predicted Nail Forces and Floor Deflections

In order to ascertain the differences in predicted nail forces and floor deflections between linear and nonlinear analyses, we selected nine floors in each of the four material groups for further study. The nine floors chosen were: 1, 3, 5, 7, 9, 11, 13, 15, and 18 for each material group. Because nail forces were not reported (Vanderbilt and others 1983), each of the nine floors of each group was reanalyzed using the linear analysis option of NONFLO. All nails were assigned a stiffness of 30,000 lb/in., the same value used in Vanderbilt and others (1983).

Table 2 presents comparisons of the maximum nail force and maximum deflection of the controlling joists as predicted from linear and nonlinear analyses of the selected floors of the four material groups. Floor 1 in all four material groups had the lowest failure load, yet the predicted maximum nail force in each of the controlling joists is high even at these relatively low failure loads. For example, the controlling joist of Floor 1 of the southern pine as-graded material had a maximum nail force of 394 pounds. At higher failure loads, the nail forces computed from the linear analysis are physically impossible to attain. While no experimental verification has been made, the nail forces predicted from the nonlinear analysis are more plausible, particularly for the floors in the lower percentiles of the CDF. In most instances the maximum nail force computed from a nonlinear analysis of a given floor occurred in the joist that failed first, or the controlling joist.

Table 2.—Comparison of maximum nail forces and maximum deflections in controlling Douglas-fir and southern pine joists as predicted from linear and nonlinear analyses of selected floors

Floor number	Maximum nail force from linear analysis, P_L	Maximum nail force from nonlinear analysis, P_{NL}	$\frac{P_{NL}}{P_L}$	Maximum deflection from linear analysis, D_L	Maximum deflection from nonlinear analysis, D_{NL}	$\frac{D_{NL}}{D_L}$
	Lb	Lb		In.	In.	
			SOUTHERN PINE (AS-GRADED)			
1	394	181	0.459	0.739	0.752	1.018
3	650	238	.366	1.184	1.213	1.025
5	343	215	.627	.617	.640	1.037
7	595	241	.405	1.149	1.204	1.048
9	516	220	.426	.908	.921	1.014
11	678	276	.407	1.386	1.529	1.103
13	826	280	.339	1.513	1.565	1.034
15	[illegible]7	288	.344	1.620	1.746	1.078
18	1,020	351	.344	1.956	2.063	1.055
			SOUTHERN PINE (ON-GRADE)			
1	372	198	.534	.762	.816	1.071
3	561	248	.442	1.161	1.281	1.103
5	532	221	.415	.966	.995	1.030
7	464	213	.459	.886	.932	1.052
9	405	212	.523	.810	.897	1.107
11	657	245	.373	1.225	1.257	1.026
13	641	252	.393	1.345	1.272	[1]0.946
15	923	296	.321	1.792	1.945	1.085
18	982	280	.285	([2])	1.587	([2])
			DOUGLAS-FIR (ON-GRADE)			
1	437	188	.430	.795	.788	.991
3	411	194	.472	.803	.821	1.022
5	468	205	[illegible]38	.899	.931	1.034
7	414	185	.447	.752	.762	1.013
9	709	257	.362	1.469	1.532	1.043
11	509	215	.422	.981	1.000	1.019
13	601	224	.373	1.077	1.088	1.010
15	831	264	.318	1.494	1.492	.999
18	1,137	295	.260	2.013	2.003	.995
			DOUGLAS-FIR (AS-GRADED)			
1	366	168	.459	.636	.623	.980
3	391	183	.468	.731	.735	1.006
5	529	232	.439	1.082	1.178	1.089
7	577	225	.390	1.072	1.074	1.002
9	659	245	.372	1.231	1.277	1.037
11	754	262	.347	1.482	1.553	1.048
13	673	239	.355	1.217	1.253	1.030
15	743	253	.341	1.342	1.338	.997
18	1,375	322	.234	2.509	2.595	1.034

[1]The linear and nonlinear analyses were controlled by different joists. This number has no significance.

[2]Missing data.

The comparisons of maximum deflection in the controlling joists (table 2) indicate that the linear analysis nearly always underpredicted deflections, although the differences in predicted deflections from the linear analyses compare more favorably with those from the nonlinear analyses than do the maximum nail forces from these analyses.

Substitute Connector Stiffnesses

Proposed in this section is to investigate an alternative process to the one already discussed. The substance of the proposed procedure depends on the amount of slip that exists in the nails at impending failure. Previously, a linear analysis was made using the conventional nail stiffness assigned in the service load range. Then the predicted failure load was scaled downward to account for the loss of stiffness in the floor with increasing load caused by the nonlinear behavior of the nails.

When a nonlinear analysis is completed and the floor is in equilibrium at its impending failure load, each nail has developed a certain amount of slip and a corresponding force. The slip response of this nail then puts it at some location on the slip curve specified by equation (1). We defined the secant drawn from the origin of the slip curve to the point on the curve at which the nail has responded to be the post-convergence secant stiffness, shown diagrammatically in figure 2 in its relative position to the connector stiffness normally used in linear analyses. In general, every nail in the floor has a different post-convergence secant stiffness, those near the ends of a joist generally having lower stiffnesses than those near the center of the joist for a uniformly loaded floor. Typical patterns of post-convergence secant stiffnesses for floors loaded with point loads and uniform loads are shown (Wheat and others 1983).

It is assumed in the finite-element formulation that all the connectors within any element along a joist have the same response characteristics; that is, the nails are not considered individually but in groups as they fall within the elements. Depending on the element length measured along the joist, the number of nails in any one element may be less than, equal to, or greater than one. The element length and number of rows of nails may vary along the joist so the number of nails per element is not necessarily constant along any joist.

A weighted average of the post-convergence secant stiffnesses of the nails in all elements is used to obtain a single substitute connector stiffness, K (see Appendix B). Use of K in a linear analysis results in predicted failure loads approximately equal to those from the nonlinear analysis. The substitute connector stiffness for a floor is a function of the post-convergence secant stiffness of the nails in the various elements; the contribution of the post-convergence secant stiffness of the nails in any element to the substitute connector stiffness depends on the location of the element within the floor.

Six regression equations were examined to find the most appropriate relationship between failure load, W_{NL}, predicted from the nonlinear analysis, and $\bar{K}$ for the 36 floors having southern pine joists and the 36 floors having Douglas-fir joists (Appendix B). The best fit for both species groups is provided by

$$\ell n\, W_{NL} = C_1 + C_2\, \ell n\, (K/1000) \qquad (5)$$

where C_1 and C_2 are constants which depend on which of the two species groups is selected. The correlation coefficients associated with this equation are 0.93 for southern pine floors and 0.92 for Douglas-fir floors. What is demonstrated is that there is good correlation between W_{NL} and $\bar{K}$. However, a more practical result is achieved if $\bar{K}$ can be related to either the joist stiffnesses (i.e. MOE), to the joist strengths, or to the MOR. Correlations were made of $\bar{K}$ with material properties for the 36 floors with southern pine joists (as-graded and on-grade combined) and the 36 with Douglas-fir joists (see Appendix B). The correlation coefficient between $\bar{K}$ and the average MOE for the group of southern pine floors is 0.25 and for the Douglas-fir floors is 0.10. Similarly, the correlation coefficients between $\bar{K}$ and the MOE of the controlling joist are 0.21 and 0.32 for the southern pine and Douglas-fir groups. The correlation coefficients between $\bar{K}$ and the average MOR for the southern pine floors and Douglas-fir floors are 0.40 and 0.44, respectively. Thus, the final state of the nails of a floor as represented by the substitute connector stiffness, $\bar{K}$, is not reliably predicted from the average joist stiffness, the average joist strength, nor the MOE of the first joist to rupture.

Regression analyses indicate, however, that $\bar{K}$ is satisfactorily related to the MOR of the controlling joist in a floor. For the 36 southern pine floors and the 36 Douglas-fir floors a relationship between $\bar{K}$ and the MOR of the controlling joist was found to be

$$K = e^{[C_1 + C_2 M/1000]} \qquad (6)$$

where M is the MOR of the controlling joist and C_1 and C_2 are constants depending on the species group. The correlation coefficients are 0.80 and 0.84, respectively, for the southern pine and Douglas-fir species groups. The obvious difficulty in using equation (6) is deciding on which joist of a floor will rupture first if all the MOR's are known at the outset. Of the 72 floors analyzed with the nonlinear model, 53 (or 74 pct) had failure loads controlled by the joist with the lowest MOR. Of the remaining 19, 17 were controlled by the joist with the second lowest MOR. When the joist with the second lowest MOR controlled, the differences between the lowest MOR and the second lowest MOR were generally under 300 lb/in.^{2} Because differences in MOR of the weakest joist and the controlling joist were generally small, it can be said that the substitute connector stiffness, K, when used in predicting the load which will cause the first occurrence of joist rupture, correlates better with the MOR of the weakest joist than with any other single input quantity.

Equation (5) should not be used to estimate the failure load once $\bar{K}$ is known because this expression is more sensitive than the finite-element analysis to small variations in $\bar{K}$. As an example, consider Floor 8 of the Douglas-fir (as-graded) group. The failure load predicted from the nonlinear analysis is 101.3 lb/ft^2. The substitute connector stiffness computed from equation (6) is 10,162 lb/in., and using this value in equation (5) predicts a failure load of 111.9 lb/ft^2. By contrast, using the same value of K in a linear finite-element analysis gives a predicted failure load of 101.5 lb/ft^2, closer to the prediction from the nonlinear analysis.

Discussion

This study was a theoretical analysis based on an assumed virgin load-slip function of the nails. The use of this particular nail slip curve in a nonlinear finite-element analysis results in a computed decrease in floor stiffness with increasing load which has been verified by experimental studies (Wheat and others 1983). This load-slip curve was chosen because its parameters, which are functions of the wood species used in the nailed joint, were available for the joist and sheathing material combinations of test floors (Wheat and others 1983).

There have been no reported experimental verifications of predicted nail slips in wood-joist floor systems. In Wheat and others (1983) comparisons are made of slip measurements with those predicted from NONFLO, but not enough measurements were taken to make any conclusions. There is a definite need to obtain reliable slip measurements throughout wood-joist floor systems and to compare these with predicted values.

The results presented here are derived from analyses of a floor system which, for a given species group, had a single configuration. The specific floor geometry chosen is often used in construction, but to extend the results of this study to a floor having a different joist spacing or mixed boundary conditions would be imprudent. Similarly, an important construction practice in building wood-joist floors is the inclusion of underlayment, perhaps particleboard, over the plywood layer. This type of construction also may warrant further theoretical and experimental scrutiny.

The floor load in this study is assumed to be uniformly distributed. Floors subjected to concentrated loads also should be examined because the pattern of nail stiffnesses at impending failure will be considerably different from the uniform load situation (Wheat and others 1983). Therefore, differences in predicted failure loads between the linear analysis, using the nail stiffness commonly assigned to the service load range, and the predicted loads from the nonlinear analysis, might take on different characteristics from the uniform load cases reported here.

An indication of how the failure loads from the various finite-element analyses compare is given in table 3. This table shows the predicted failure loads for selected floors of both southern pine and Douglas-fir material groups as computed from the following:

1. A linear analysis using a constant connector stiffness, K_L, of 30,000 lb/in.
2. A nonlinear analysis in which equation (1) is the assumed force-slip relationship for the connectors.
3. A linear analysis in which the connector stiffness is K as determined from equation (6).

The results are given for floors at various levels of the CDF. For these floors, using $\bar{K}$ provides very satisfactory predictions of failure load when compared with the results of the nonlinear analyses.

Table 3.—Predicted failure loads from various analyses of selected floors with as-graded joists

Floor number	Predicted failure load			$\bar{K}$, inch/inch (eq. 6)	Percentile on cumulative distribution function
	W_L[1]	W_{NL}	$W_{\bar{K}}$		
	---- Lb/ft² ----				
			SOUTHERN PINE		
1	64.0	59.9	59.9	13,250	0.47
3	100.8	91.0	91.9	11,000	2.34
5	112.7	108.4	108.9	11,250	4.21
15	180.5	170.2	173.5	8,920	50.00
18	296.4	269.5	279.5	7,900	↓.86
			DOUGLAS-FIR		
1	79.2	72.3	72.1	12,320	.36
3	86.9	80.9	81.3	12,940	1.81
5	98.1	94.4	95.9	11,860	3.26
15	170.8	152.5	154.0	9,140	50.36
18	265.9	235.0	240.7	6,980	95.29

[1]W_L = failure load from linear analysis,
K = 30,000 lb/in.
W_{NL} = failure load from nonlinear analysis.
W_K = failure load from linear analysis, K = $\bar{K}$.

Conclusions

The main thrust of this study was a comparison of failure loads of wood-joist floors as predicted by a linear finite-element program FEAFLO and a nonlinear finite-element program NONFLO in which the characteristically nonlinear load-slip behavior of the nails was included as the only nonlinearity. The simple polynomial expressions derived in this study relate the failure loads from the NONFLO analysis to those from FEAFLO, thus enabling one to use FEAFLO to predict failure loads. This is accomplished by simply scaling results of a FEAFLO analysis by a reduction factor. The overall average reduction factor is 0.91 for the floor configuration used in this study. A nonlinear program such as NONFLO could be used with sample data to compute a similar factor for any floor design.

The alternate procedure introduced in this study relies upon the calculation of a substitute connector stiffness, defined as the single value of nail stiffness which, when used in a linear analysis of a floor such as FEAFLO, will result in near-equivalent predictions of failure load as determined from the more costly nonlinear (NONFLO) analysis. The substitute connector stiffness is closely related to the MOR of the controlling joist. Furthermore, the controlling joist is theoretically likely to be the one with the lowest MOR in the floor system. The relationship between K and the MOR of the controlling joist, M, is

$$K = e^{(C_1 + C_2 M/1000)}$$

where C_1 and C_2 are constants given in table B-4.

With M obtainable from input data, K can be calculated and used in FEAFLO to predict the failure load. Unlike the first procedure, no further scaling of the failure load is necessary to account for the nonlinearity of the nails.

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Appendix A
Predicted Failure Loads

Table A-1.—Predicted failure loads from linear and nonlinear analyses for floors with southern pine joists

Floor number	Failure load from FEAFLO, W_L	Failure load from NONFLO, W_{NL}	$\frac{W_{NL}}{W_L}$	Modulus of rupture of controlling joist
	——— Lb/ft² ———			Lb/in.²
		AS-GRADED		
1	64.0	59.9	0.936	1090 (9)[1]
2	76.9	72.3	.940	1378 (2)
3	100.8	91.0	.903	1944 (9)
4	109.9	102.7	.935	1639 (9)
5	112.7	108.4	.962	1841 (10)
6	112.7	105.6	.937	2212 (8)
7	113.8	106.1	.932	2004 (5)
8	115.9	105.9	.914	2179 (3)
9	117.1	109.4	.934	2447 (3)
10	120.9	110.5	.914	2820 (9)
11	123.5	117.1	.948	1780 (2)
12	131.3	121.0	.922	2419 (6)
13	134.8	120.8	.896	2491 (9)
14	149.7	134.0	.895	3060 (3)
15	180.5	170.2	.943	2904 (6)
16	213.2	188.9	.886	4420 (8)
17	255.0	235.2	.922	4711 (3)
18	296.4	269.5	.909	3464 (1)
		ON-GRADE		
1	70.8	69.3	.979	1090 (6)[1]
2	92.0	84.8	.922	1787 (3)
3	92.3	88.9	.963	1561 (8)
4	97.1	91.2	.939	1392 (4)
5	102.7	95.2	.927	2092 (8)
6	103.3	99.5	.963	1649 (6)
7	104.2	100.7	.966	1648 (8)
8	105.2	96.1	.913	2419 (5)
9	106.6	105.0	.985	1464 (10)
10	110.1	97.6	.886	2224 (7)
11	112.5	100.0	.889	1989 (7)
12	125.6	117.3	.934	1796 (7)
13	132.9	128.7	.968	1970 (8)[2]
14	141.6	128.0	.904	2929 (9)
15	165.1	150.8	.913	2891 (4)
16	188.4	173.6	.921	3276 (7)
17	224.2	198.4	.885	4192 (2)
18	269.0	245.9	.914	5226 (5)

[1]Numbers in parentheses indicate controlling joist (see fig. 1 for joist numbering).

[2]Joist 6 controlled in the linear analysis.

Table A-2.—Predicted failure loads from linear and nonlinear analyses for floors with Douglas-fir joists

Floor number	Failure load from FEAFLO, W_L	Failure load from NONFLO, W_{NL}	$\frac{W_{NL}}{W_L}$	Modulus of rupture of controlling joist
	——— Lb/ft² ———			Lb/in.²
		AS-GRADED		
1	79.2	72.3	0.913	1592 (9)[1]
2	79.9	73.5	.920	1716 (9)
3	86.9	80.9	.931	1393 (7)
4	91.8	82.2	.895	1694 (7)
5	98.1	94.4	.962	1747 (8)
6	103.9	94.4	.909	2480 (3)
7	108.7	97.1	.893	1902 (7)
8	110.6	101.3	.916	2371 (9)
9	124.2	113.2	.911	2437 (2)
10	127.1	111.4	.876	2184 (3)
11	130.6	119.0	.911	2418 (3)
12	137.8	123.0	.893	2712 (2)
13	142.3	130.4	.916	3115 (2)
14	149.1	128.7	.863	2671 (5)
15	170.8	152.5	.893	2799 (4)
16	192.5	168.5	.875	3473 (3)
17	225.1	197.2	.876	3955 (3)
18	265.9	235.0	.884	5429 (9)
		ON-GRADE		
1	69.0	61.7	.894	3145 (9)[1]
2	73.9	69.6	.942	1414 (5)
3	75.9	70.2	.925	1164 (4)
4	81.1	72.7	.896	1601 (5)
5	82.8	76.7	.926	1716 (4)
6	84.0	79.1	.942	1439 (4)
7	87.8	81.5	.928	1934 (6)
8	92.0	83.8	.911	1285 (9)
9	98.7	87.5	.887	1456 (3)
10	100.4	91.9	.915	1568 (8)
11	103.5	94.9	.917	1555 (5)
12	114.7	111.2	.969	1987 (8)
13	119.3	107.4	.900	2851 (7)
14	140.9	127.6	.906	1917 (7)
15	159.8	139.5	.873	3036 (3)
16	177.8	158.6	.892	3201 (6)
17	200.9	190.8	.950	2754 (6)
18	230.8	201.2	.872	3860 (9)

[1]Numbers in parentheses indicate controlling joist (see fig. 1 for joist numbering).

Appendix B
Substitute Connector Stiffness

In the main body of this report the post-convergence secant stiffness for the nails in any element is defined as the nail stiffness in an element after the floor comes into equilibrium at impending failure. The nails throughout the floor generally have different magnitudes of post-convergence secant stiffness when equilibrium is achieved and the analysis has converged. The substitute connector stiffness is calculated from this final state of the nails.

First a weighted average nail secant stiffness along a single joist is computed as follows:

$$K_j = \frac{\sum_{i=1}^{NE} (K_e n_e L_e)}{\sum_{i=1}^{NE} (n_e L_e)}$$

in which the summation is taken over the number of elements, NE, in a joist, and

K_e = post-convergence secant stiffness for the nails of element i of joist j

L_e = the length of element i of joist j

n_e = the number of rows of nails in element i of joist j

The substitute connector stiffness for the floor is *defined* as

$$\bar{K} = \frac{\sum_{j=1}^{NJ-2} K_j}{NJ - 2}$$

or as simply the average K_j for those joists believed to be away from the direct influence of the simply supported edges of the floor. For all the floors of this study, the number of joists free to deflect, NJ, is 10 *and* the first joist on either side of the floor is eliminated in the calculation of $\bar{K}$.

The substitute connector stiffness thus is seen to be a single value of nail stiffness which represents the weighted average nail stiffness throughout the floor. It is calculated at the completion of a nonlinear analysis when convergence has been attained. A procedure which seeks to predict a proper value of $\bar{K}$ is described later in this appendix.

A demonstration of the accuracy of using the $\bar{K}$ values to predict failure loads is shown in tables B-1 and B-2 for the four material groups. Nine selected floors along the CDF were chosen for analysis. The comparisons of linear and nonlinear analysis results are good and show that the failure loads, W_{NL}, predicted from the nonlinear analyses generally *are smaller than the failure loads,* $W_{\bar{K}}$, *predicted* from the linear analyses using $\bar{K}$. The differences are less than 2 percent. The maximum deflections (not shown in tables) of the controlling joists at impending failure predicted from the nonlinear analyses are generally 1 to 2 *percent greater than the deflections predicted from the* linear analyses using $\bar{K}$.

Table B-1.—Comparison of predicted failure loads for southern pine joists from a nonlinear analysis and from a linear analysis using $\bar{K}$ as the nail stiffness

Floor number	Failure load from nonlinear analysis W_{NL}	$\bar{K}$	Failure load from linear analysis using $\bar{K}$ $W_{\bar{K}}$	$W_{NL}/W_{\bar{K}}$
	Lb/ft²	Lb/in.	Lb/ft²	
		AS-GRADED		
1	59.9	16,400	60.8	0.985
3	91.0	12,100	92.6	.983
5	108.4	12,400	109.4	.991
7	106.1	11,200	107.5	.987
9	109.4	12,300	110.6	.989
11	117.1	10,000	118.0	.992
13	120.8	7,700	120.8	1.000
15	170.2	7,400	171.7	.991
18	269.5	4,600	272.3	.990
				[1].990
				[2].005
		ON-GRADE		
1	69.3	13,500	70.2	.987
3	88.9	12,000	89.9	.989
5	95.2	11,800	96.1	.991
7	100.7	12,200	101.2	.995
9	105.0	9,600	106.4	.987
11	100.0	10,200	102.0	.995
13	128.7	8,200	129.3	.980
15	150.8	7,300	152.5	.989
18	245.9	6,800	247.8	.989
				[1].989
				[2].005

[1]Mean.

[2]Standard deviation.

Correlation of Substitute Connector Stiffness and Failure Load

Each of the 72 floors chosen for this study was analyzed using NONFLO to determine its substitute connector stiffness. The results were used *to plot the failure load,* W_{NL}, versus $\bar{K}$ for the 36 floors with southern pine joists and the 36 floors with Douglas-fir joists as shown in figure B-1. Because of the variability in material properties, both of these plots contain some scatter. These plots would have no scatter *if there were no variability* (Wheat and others 1983) that is, if all the joists in any floor were assigned the same stiffness and strength. Table B-3 shows the results of fitting six functions to the data in each plot in figure B-1, and based on the correlation coefficients, R, the best fit for both the southern pine and Douglas-fir data is obtained with

$$\ln W_{NL} = C_1 + C_2 \ln (\bar{K}/1000)$$

where C_1 and C_2 are constants as tabulated. The resulting correlation coefficient using this equation is 0.93 for the southern pine floors and 0.92 for the Douglas-fir floors.

Table B-2.—Comparison of predicted failure loads for Douglas-fir joists from a nonlinear analysis and from a linear analysis using K as the nail stiffness

Floor number	Failure load from nonlinear analysis W_{NL}	K	Failure load from linear analysis using K W_K	$\frac{W_{NL}}{W_K}$
	Lb/ft²	Lb/in	Lb/ft²	
		AS-GRADED		
1	72.3	14,700	72.8	0.993
3	80.9	15,000	82.0	.987
5	94.4	10,700	95.3	.991
7	97.1	11,900	98.8	.983
9	113.2	10,400	114.3	.990
11	119.0	8,100	120.0	.992
13	130.4	9,600	130.9	.996
15	152.5	9,900	154.7	.986
18	235.0	4,900	236.3	.995
				[1].990
				[2].004
		ON-GRADE		
1	61.7	15,800	63.1	.978
3	70.2	13,900	71.2	.986
5	76.7	12,300	77.7	.987
7	81.5	13,400	82.6	.987
9	87.5	10,100	89.1	.982
11	94.9	10,700	95.6	.993
13	107.4	10,800	109.2	.984
15	139.5	8,700	141.6	.985
18	201.2	7,000	203.2	.990
				[1].990
				[2].005

[1]Mean.

[2]Standard deviation

Thus, an acceptable correlation exists between the failure load and the substitute connector stiffness; furthermore, the previous section shows that using the substitute connector stiffness in the linear finite-element analysis results in a failure load close to that predicted from a nonlinear analysis.

Correlation of Substitute Connector Stiffness with Joist Stiffness and Strength Properties

This brief discussion describes attempts to predict the substitute connector stiffness from the input parameters of the finite-element computer programs, namely the joist stiffnesses and strengths. For the 36 floors with southern pine joists and 36 with Douglas-fir joists, the substitute connector stiffnesses were plotted against the following:

1. The average MOE of the middle eight joists.
2. The MOE of the joist that ruptured.
3. The average MOR of the middle eight joists.
4. The MOR of the joist that ruptured.

Figure B-2 plots K against the average MOE and MOR of the middle eight interior joists and the MOE and MOR of the controlling joist for the southern pine and Douglas-fir material groups. Both as-graded and on-grade data are shown.

For average MOE, the correlation coefficients, R, obtained for the two plots with all 36 data points included are 0.25 for the southern pine group and 0.10 for the Douglas-fir group, indicating there is not a good correlation between K and the average MOE. Therefore, K cannot be predicted reliably from the average joist stiffness.

For the plot of K against the MOE of the controlling joist for the southern pine and Douglas-fir material groups, the correlation coefficients for the two sets of data are 0.21 and 0.32, respectively. Again the correlations are poor.

In a similar fashion, the plots of K versus average MOR of the middle eight interior joists for the southern pine and Douglas-fir groups indicate no real correlation, meaning that K is also not predictable from the average joist strength. The R values for the floors having southern pine and Douglas-fir joists in these correlations are 0.40 and 0.44, respectively.

For a given floor, the final state of the nails (or the substitute stiffness) at the point of occurrence of the first joist rupture is not predictable from the average value of either of the two material properties that have the greatest influence on the behavior of the floors. Nor is it predictable from the MOE of the controlling joist. So, although there seems to be a reasonable correlation between the failure load and the post-convergence nail stiffnesses, there is no way to predetermine, based on the above material properties, what substitute connector stiffness is appropriate for use in predicting the failure load.

More success is achieved when $\bar{K}$ is correlated with the MOR of the controlling joist, where the data for the 36 floors with southern pine joists and the 36 with Douglas-fir joists are plotted in figure B-2. Three regression equations were fitted to each set of data, as shown in table B-4. The best correlation of $\bar{K}$ with the MOR of the controlling joist for the southern pine floors is obtained with

$$\ln \bar{K} = C_1 + C_2 \ln (MOR/1000)$$

The Douglas-fir data are best represented by

$$\bar{K} = e^{C_1 + C_2 (MOR/1000)}.$$

However, the latter expression is suitable for both species groups.

The immediate practical difficulty is how does one know initially which joist will control, assuming the joist MOR values are given? As discussed in the body of this report, it is most likely that the joist with the lowest MOR will control. This was noted from the nonlinear finite-element analyses of the 72 floors and does not represent an experimentally verified observation.

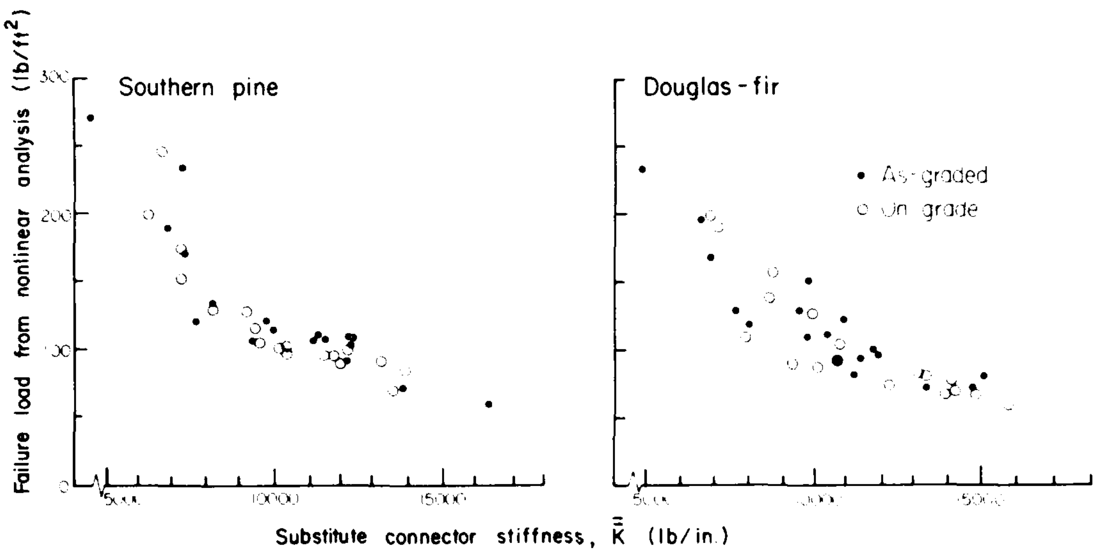

Figure B-1.—Failure load from nonlinear analysis versus substitute connector stiffness for floors with southern pine and Douglas-fir joists. (ML83 5451)

Table B-3.—Regression equations for failure load versus $\bar{K}$

Function[1]	Southern pine			Douglas-fir		
	C_1	C_2	R	C_1	C_2	R
$\ln W_{NL} = C_1 + C_2 \ln(\bar{K}/1000)$	7.43	−1.17	0.93	7.38	−1.16	0.92
$\ln W_{NL} = C_1 + C_2 (\bar{K}/1000)$	6.00	−.12	.91	5.89	−.11	.91
$\ln W_{NL} = C_1 + C_2 (e^{K/1000})$	4.80	$-.62 \times 10^{-7}$	.40	4.75	$-.14 \times 10^{6}$	.54
$W_{NL} = C_1 \ln(\bar{K}/1000) + C_2$	−161.54	495.01	.90	−142.24	446.27	.90
$W_{NL} = C_1 (\bar{K}/1000) + C_2$	−16.18	290.52	.86	−13.61	259.61	.87
$W_{NL} = C_1 e (\bar{K}/1000) + C_2$	$-.62 \times 10^{-5}$	128.23	.28	$-.14 \times 10^{4}$	121.84	.87

[1] W_{NL} = failure load predicted from nonlinear analysis.
$\bar{K}$ = substitute connector stiffness.

Table B-4.—Regression equations for $\bar{K}$ versus modulus of rupture of controlling joist

Function	Southern pine			Douglas-fir		
	C_1	C_2	R	C_1	C_2	R
$\bar{K} = e^{C_1 + C_2 MOR/1000}$	9.730	−0.218	0.80	9.812	−0.247	0.84
$\ln\bar{K} = C_1 + C_2 e^{MOR/1000}$	9.292	−.00426	.56	9.332	−.00460	.64
$\ln\bar{K} = C_1 + C_2 \ln(MOR/1000)$	9.669	−.577	.82	9.711	−.612	.83

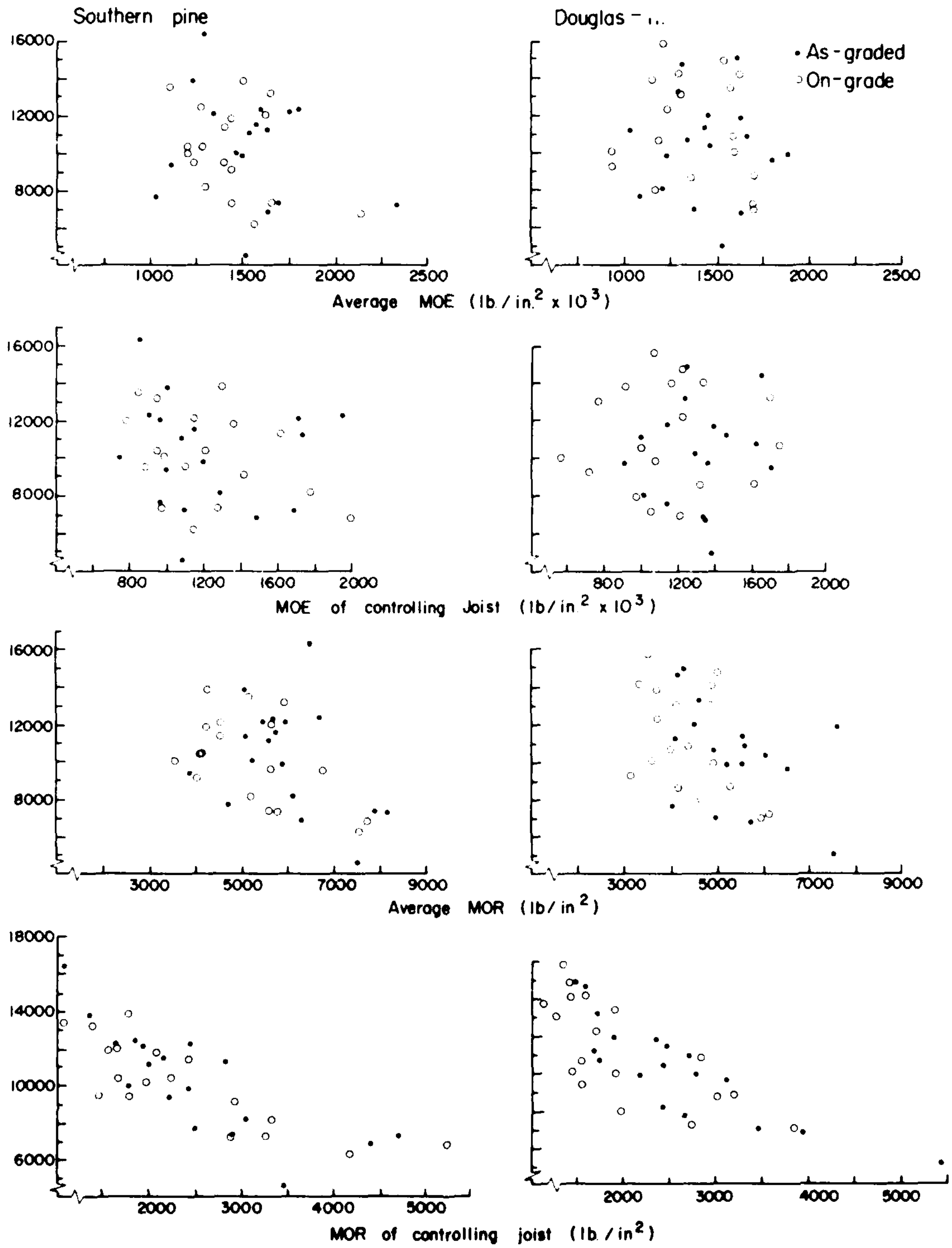

Figure B-2.—Substitute connector stiffness ($\bar{K}$) versus average modulus of elasticity (MOE), MOE of controlling joist, average modulus of rupture (MOR), and MOR of controlling joist for floors with southern pine and Douglas-fir joists. (ML83 5452)

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